PART THREE

Design

Military, semipermanent, nonstandard fixed highway bridges are designed for a given MLC. Simply supported stringer bridges are recommended since they are easy to design and construct. The materials available and the capabilities of the construction unit must be known before conducting the design process. The design should be economical in materials and construction effort but should not require excessive maintenance.

Chapter 6

Bridge Superstructures

Steel is not always readily available and requires special equipment for its use in construction. However, steel stringers are preferred over timber stringers because of their strength and capability for supporting longer spans. Use steel stringers for TO nonstandard fixed bridges whenever possible. The deck should normally be plank or laminated timber. The wearing surface should be either timber planks or asphalt. Use a concrete deck if concrete is available and a more durable structure is desired.

DESIGN PHASES

6-1. Design is a two-phase process. The first phase involves determining the design loads and their effects in terms of moment and shear forces. The second phase involves selecting members that have sufficient strength to resist the effects of the intended loads on the bridge. Before considering the design process complete, the failure modes (lateral buckling, excessive deflection, end bearing, and so forth) as well as moment and shear must be checked.

DESIGN SEQUENCE

6-2. A logical design sequence is necessary to prevent design omissions and to eliminate unnecessary effort. Select and design members and accessories to prevent any of the five modes of beam failure (excessive vertical deflection, bending, shear, lateral deflection, and bearing).

6-3. The superstructure design sequence is discussed below and includes beam failure as part of the design process. The first nine steps are discussed in this chapter, and *step 10* is discussed in *Chapter 9*.

Step 1. Perform a reconnaissance of the bridge site and determine the bridge's requirements.

Step 2. Determine the number of stringers.

Step 3. Design the deck.

Step 4. Design the stringers.

Step 5. Check the vertical deflection.

Step 6. Design the lateral bracing.

Step 7. Check the dead load.

Step 8. Check the shear forces.

Step 9. Design the end-bearing components.

Step 10. Design the connections.

RECONNAISSANCE

6-4. Perform a reconnaissance of the bridge site as outlined in *Chapter 2*. Before proceeding with the bridge design, determine the—

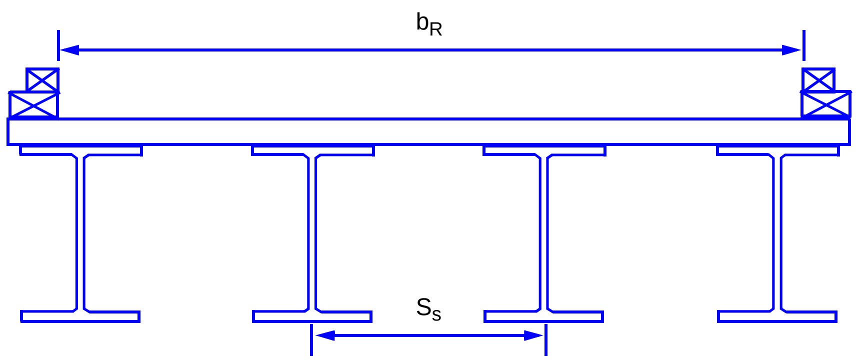
- Span length of the bridge (in feet).
- Design classification (wheeled, tracked, or both).
- Number of lanes.
- Available construction materials.
- Equipment and personnel required.
- Site constraints.

6-5. After determining the specifications, design the superstructure of the bridge as discussed below. Design the substructure according to the procedures outlined in *Chapter 7*.

NUMBER OF STRINGERS DETERMINATION

6-6. After determining the bridge specifications, estimate the minimum number of stringers required for a given span length. The total number of stringers depends on the moment capacity of an individual stringer, the roadway width, and the center-to-center stringer spacing. The goal is to produce the most economical, safe bridge design using the least number of stringers possible.

6-7. Determine the minimum number of stringers by using the maximum center-to-center stringer spacing and the roadway width *(Figure 6-1)*. Stringer spacing should not exceed 6 feet for timber-deck bridges and 8 feet for concrete-deck bridges. Use *Table 6-1* to determine the number of stringers required according to the bridge classification, the roadway width, and the deck type.



LEGEND:
b_R = roadway width
S_s = center-to-center stringer spacing
(≤6 ft for timber decks and ≤8 ft for concrete decks)

Figure 6-1. Stringer Spacing

Table 6-1. Number of Stringers Required

Bridge Classification	One Traffic Lane			Two or More Traffic Lanes		
	Curb-to-Curb Roadway Width (b_R)	Timber Deck	Concrete Deck	Curb-to-Curb Roadway Width (b_R)	Timber Deck	Concrete Deck
4 to 12	9 ft 0 in	4	4	18 ft	4	4
13 to 30	11 ft 0 in	4	4	18 ft	4	4
31 to 60	13 ft 2 in	4	4	24 ft	5	4
61 to 100	14 ft 9 in	4	4	27 ft	6	5
101 to 150	16 ft 5 in	4	4	32 ft	7	5

6-8. The number of stringers required can also be determined if the roadway width and the desired stringer spacing is known. Compute as follows:

$$N_s = \frac{b_R}{S_s} + 1 \qquad (6\text{-}1)$$

where—

N_S = *total number of stringers, raised to the next higher whole number if a noninteger*

b_R = *curb-to-curb roadway width, in feet*

S_S = *center-to-center stringer spacing, in feet*

6-9. After finding the number of stringers required (by either method), compute the actual stringer spacing as follows:

$$S_s = \frac{b_R}{N_s - 1} \tag{6-2}$$

where—

S_S = *actual center-to-center stringer spacing, in feet*

b_R = *curb-to-curb roadway width, in feet*

N_S = *total number of stringers, raised to the next higher whole number if a noninteger (equation 6-1)*

DECK DESIGN

6-10. The deck system includes the structural deck, the wearing surface, and the curb and handrail systems. The deck of a stringer bridge supports the vehicles and distributes the load to the stringers. In a deck design (for either timber or concrete decks), the effective span length over which the loads are distributed must be known. Use this measurement to compute the dead load that is supported by the stringers.

6-11. For a deck supported on timber stringers, first compute for the clear distance between the support stringers *(equation 6-3)* and then compute for the effective span length *(equation 6-4)*.

$$L_c = S_s - \frac{t_s}{12} \tag{6-3}$$

where—

L_c = *clear distance between supporting stringers, in feet*

S_S = *actual center-to-center stringer spacing, in feet (equation 6-2)*

t_S = *thickness of stringer, in inches (Table D-2, pages D-3 and D-4)*

$$S_{eff} = L_c + \frac{t_s}{24} \tag{6-4}$$

where—

S_{eff} = *effective span length, in feet*

L_c = *clear distance between supporting stringers, in feet (equation 6-3)*

t_S = *thickness of stringer, in inches*

6-12. For timber stringers, a standard stringer nominal width of 4 inches is suggested for initial design calculations. The effective span length should not exceed the distance between the edges of the top flange of the supporting stringers plus the thickness of the deck. If the deck is supported on steel stringers, compute for the distance between the edges of the top flange of the support stringers *(equation 6-5)* and then for the effective span length *(equation 6-6)*.

$$L_e = S_s - \frac{b_f}{12} \tag{6-5}$$

where—

L_e = *distance between edges of top flange of supporting stringers, in feet*
S_S = *actual center-to-center stringer spacing, in feet (equation 6-2)*
b_f = *stringer-flange width, in inches*

$$S_{eff} = L_e + \frac{b_f}{24} \quad (6\text{-}6)$$

where—

S_{eff} = *effective span length, in feet*
L_e = *distance between edges of top flange of supporting stringers, in feet (equation 6-5)*
b_f = *stringer-flange width, in inches*

6-13. For steel stringers, assume an initial flange width of 12 inches for the deck computations. The effective span length should not exceed the distance between the edges of the top flange of the supporting stringers plus the thickness of the deck.

Timber Deck

6-14. Timber decks are constructed with the long dimension of the planks placed either horizontally (flat) (plank deck) or vertically (on edge) (laminated deck). The vertically oriented planks of a laminated deck are nailed to each other. *Figure 6-2* shows a sketch of both timber-deck orientations. Install a wearing surface to prevent wear. On timber-deck bridges, the wearing surface should consist of a 2- or 3-inch-thick timber treadway. On one-lane bridges, the treadway should be limited to the path of the wheels or tracks. On two-lane bridges, the treadway should fully cover the deck. Place the treadway between the curbs rather than under the curbs.

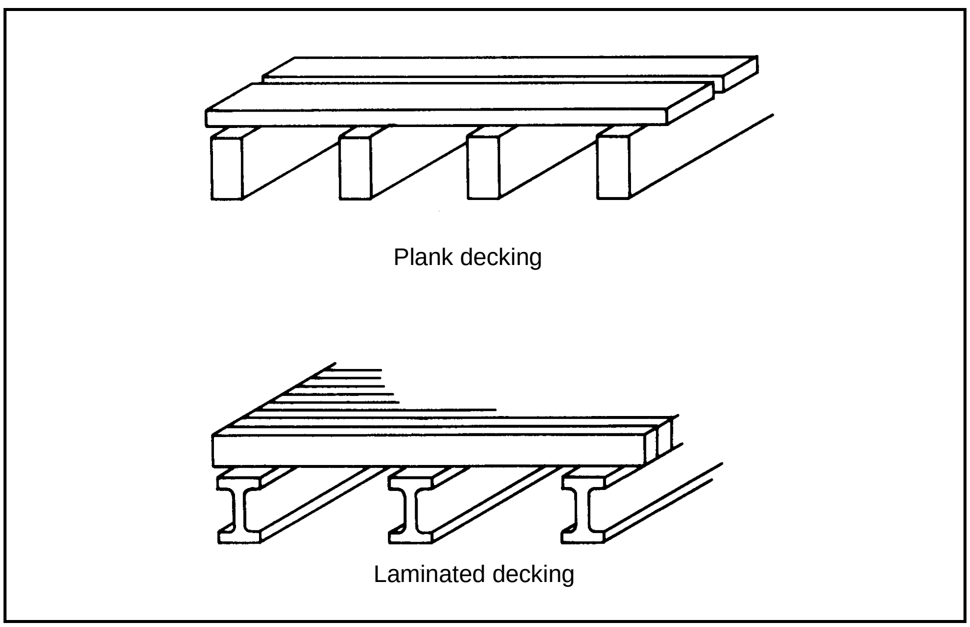


Figure 6-2. Timber-Deck Orientations

Plank Deck

6-15. A plank deck is the simplest to design and construct. It consists of a series of sawn lumber planks placed flatwise across supporting beams. Each plank is normally 10 or 12 inches wide and 4 inches thick. Plank decks are used primarily on low-volume or special-use roads. They are not suitable for asphalt pavement because of large live-load deflections and movements from moisture changes in the planks.

6-16. Determine the required deck thickness by using *Figure 6-3*. Plot the effective span length *(equation 6-4)* and the desired MLC. The minimum deck thickness is 3 inches. Use a laminated deck when the required deck thickness exceeds 6 inches. Use deck planks made from dimensioned lumber having a thickness equal to or greater than the required deck thickness. If the available deck material is not thick enough, layer planks until achieving the required thickness plus 2 inches. The extra 2 inches will compensate for the structural inefficiencies of layered planks.

6-17. Normally, install the decking in a perpendicular direction to the bridge's centerline for ease and speed of construction. Install the decking with a 1/4-inch space between the planks for expansion, better drainage, and air circulation.

Laminated Deck

6-18. Large stringer spacing and high design classifications usually require a thicker decking (laminated decks are more economical in this case). Although layering strengthens the plank decks, laminated decks are much stiffer.

6-19. **Required Deck Thickness.** Loads are spread out more effectively in a laminated deck than in a conventional plank deck. Lamination has the effect of shortening the effective deck span between stringers by about 25 percent. To design a laminated deck—

- Adjust the effective span length *(equation 6-6)* by multiplying its value by a factor equal to 0.75. This value will now become the adjusted effective span length.
- Determine the required deck thickness of the laminated deck from *Figure 6-3* (assuming that it equals that for a plank deck). Use the adjusted effective span length and the MLC. The minimum deck thickness required is 3 inches.

6-20. **Lamination.** The performance of live-load deflections depends on the effectiveness of the nails in transferring loads between adjacent boards. To create a laminated deck, place the planks vertically. Make sure that the deck is well-nailed or -glued to the adjacent board over the full length. Nails should be placed at a minimum of 1 1/2 inches on center along the length of the boards. The nail pattern should be staggered to prevent splitting of the lumber.

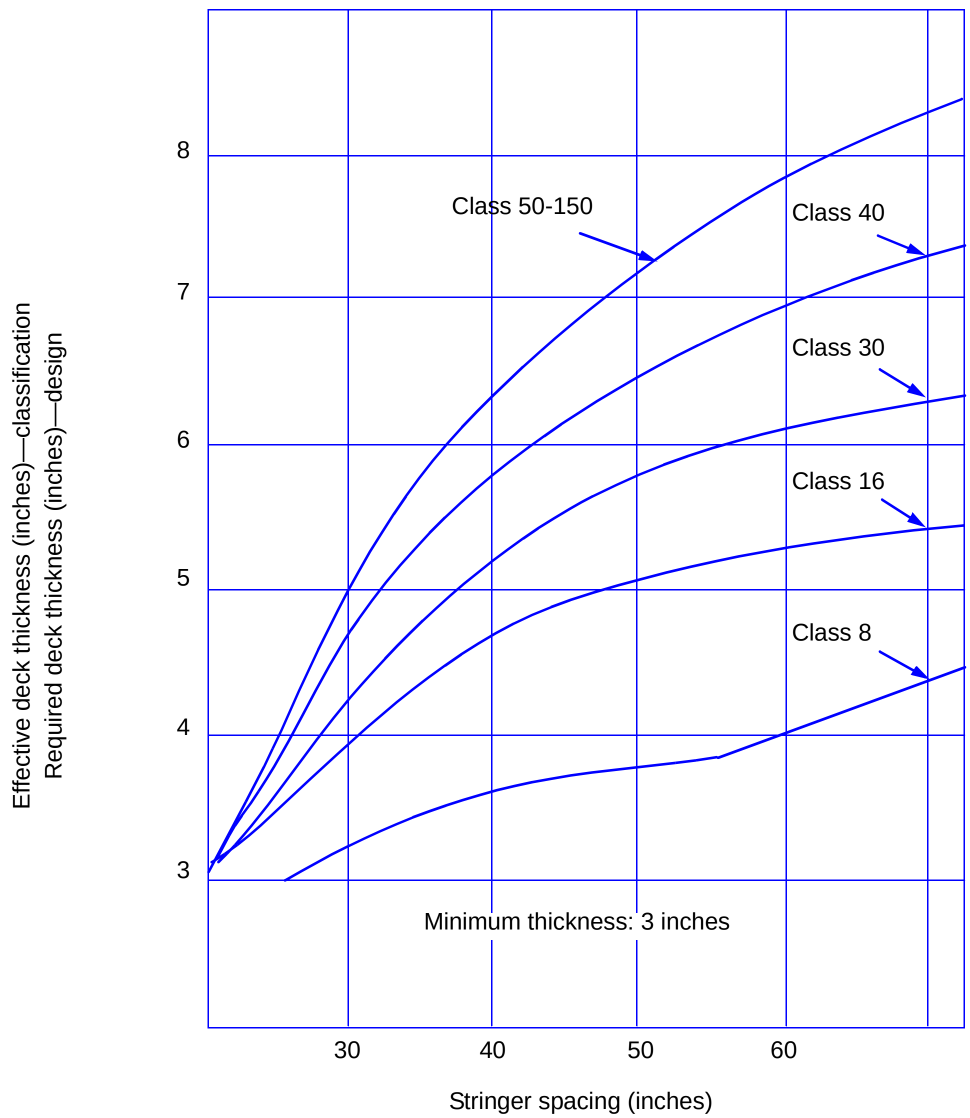


Figure 6-3. Required and Effective Deck Thicknesses (Timber Deck)

Reinforced Concrete Deck

6-21. To build a more permanent structure, use a concrete deck *(Figure 6-4, page 6-8)*. Reinforced concrete decks can span greater distances than timber decks. Concrete decks use fewer stringers, which can be spaced up to 8 feet apart. The construction process is more difficult because of the required formwork, procuring and setting of steel, placing of concrete, and curing. However, if material and time are available, use concrete decking for a stronger flooring system.

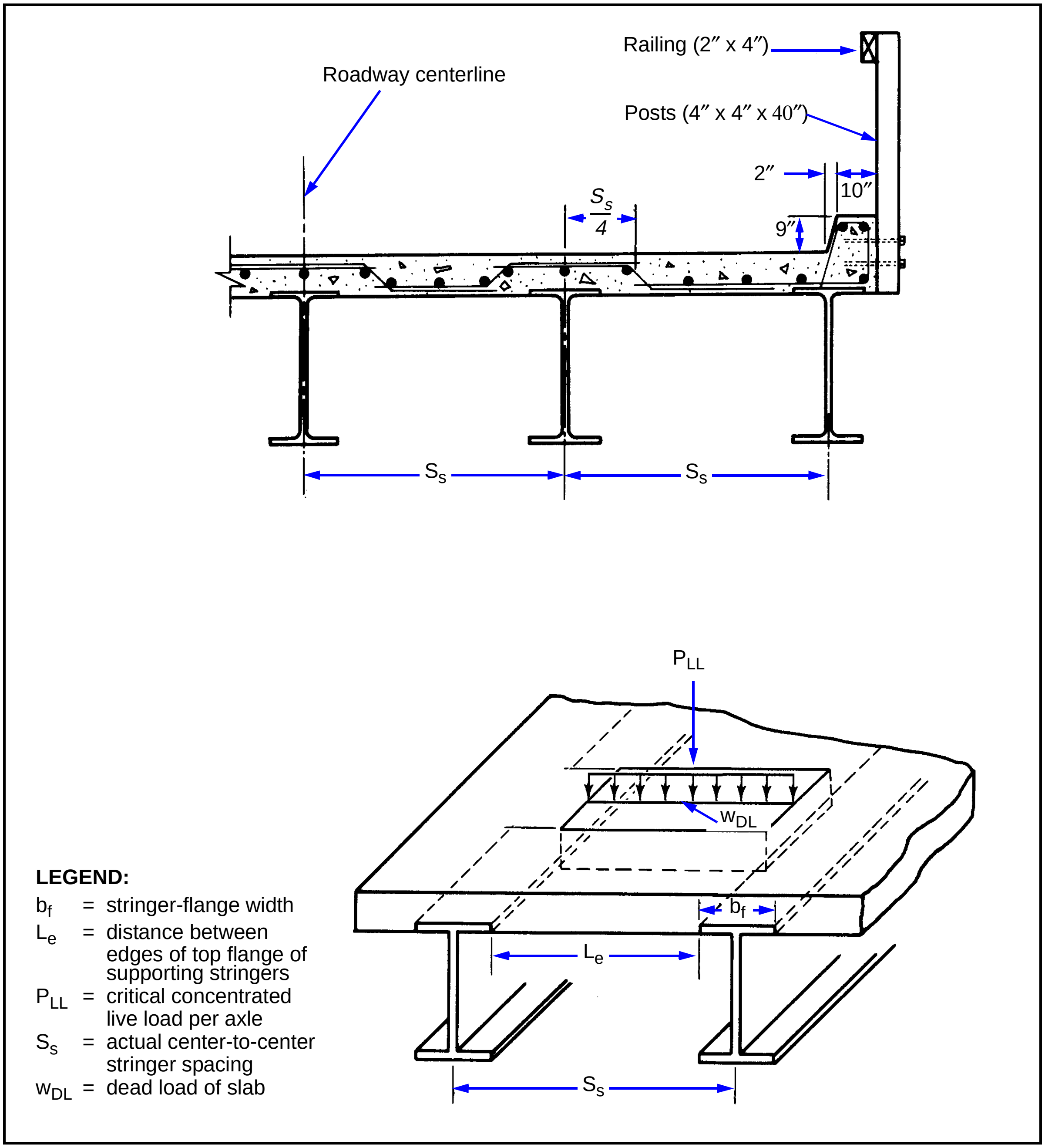


Figure 6-4. Cross Section of a Steel-Stringer Bridge With a Concrete Deck

6-22. **Concrete Compressive Strength.** In the US, the design strength of concrete corresponds to the compressive strength (in psi) of test cylinders that are 6 inches in diameter and 12 inches high and are measured on the 28th day after they are placed. *Table 6-2* lists various concrete compressive strengths used in structural design. In most situations, use a compressive strength equal to 3,000 psi for design purposes unless what will be available from the concrete source is known.

Table 6-2. Compressive Strength of Concrete

Concrete Type	f′c (in psi)
Nonprestressed	3,000 to 4,000
Prestressed	5,000 to 6,000
Special situations	6,000 to 12,000
High strength	10,000 to 15,000

6-23. **Reinforcing Steel Yield Strength.** Steel reinforcement may consist of bars, welded-wire fabric, or wires. In the US, reinforcing bars are available in sizes of 3/8 to 2 1/4 inches nominal diameter. *Table 6-3* lists various steel yield strengths for reinforcing steel used in structural design.

Table 6-3. Yield Strength of Reinforcing Steel

ASTM Designation	Grade	Bar Size No.	fy (in psi)
A615-94 (billet steel)	40	3 to 6	40,000
	60	3 to 18	60,000
A616-93 (rail steel)	50	3 to 11	50,000
	60	3 to 11	60,000
A617-93 (axle steel)	40	3 to 11	40,000
	60	3 to 11	60,000
A706-81 (low-alloy steel)	—	3 to 18	60,000

6-24. **Slab Dimensions.** Design a concrete deck as a one-way slab (continuous at both ends) with a design span equal to the effective span length *(equation 6-6)*. In designing a one-way slab, consider, for example, a typical 12-inch-wide strip. The continuous slab may be designed as a continuous beam having a known width of 12 inches; the slab thickness is now the only unknown. For design calculations, assume a slab thickness equal to 7 inches.

6-25. **Wearing Surface.** If desired, use a wearing surface such as asphalt. If asphalt is used, the wearing-surface thickness should be 1 1/2 inches.

6-26. **Dead Load.** Compute the dead-load weight of the slab by considering its own weight and any wearing surface for a 12-inch-wide strip. Compute the slab's dead-load weight and dead-load bending moment as follows:

$$w_{DL} = 1.4\left(\frac{t_s U_c + t_w U_w}{12{,}000}\right) \qquad (6\text{-}7)$$

where—

w_{DL} = *dead load of slab, in kpf of width*
t_S = *slab thickness, in inches (paragraph 6-24)*

U_c = *unit weight for concrete, in pounds per cubic foot (Table 6-4)*
t_w = *wearing-surface thickness, in inches (paragraph 6-25)*
U_w = *unit weight for wearing-surface material, in pounds per cubic foot (Table 6-4)*

$$M_{DL} = \frac{w_{DL} S_{eff}^{2}}{10} \qquad (6\text{-}8)$$

where—

M_{DL} = *dead-load bending moment of the slab, in kip-feet per foot of width*
w_{DL} = *dead load of slab, in kip-feet per foot of width (equation 6-7)*
S_{eff} = *effective span length, in feet (equation 6-4)*

Table 6-4. Unit Weights for a Dead-Load Computation

Material	Unit Weight (lb/ft^3)
Steel or cast steel	490
Cast iron	450
Aluminum alloys	175
Timber (treated or untreated)	50
Concrete (plain or reinforced)	150
Compacted sand, earth, gravel, or ballast	120
Loose sand, earth, or gravel	100
Macadam or gravel (rolled)	140
Cinder filling	60
Asphalt pavement	150
Railway rails, guard rails, and fastenings (per linear foot of track)	200
Stone masonry	170

6-27. **Live Load.** Determine the live-load bending moment acting on a 12-inch-wide strip of slab with reinforcement perpendicular to the traffic. The live load is that for the desired wheeled vehicle *(Appendix B)* with the tire load positioned so that it produces the most critical loading between the stringers.

6-28. Compute the critical, concentrated live load per wheel used for design and the live-load moment for the slabs as follows:

$$P_{LL} = P_{max} \qquad (6\text{-}9)$$

where—

P_{LL} = *critical concentrated live load per wheel, in kips*
P_{max} = *maximum single-axle load, in tons (Table B-1, pages B-2 through B-5, Column 4)*

and—

$$M_{LL} = 1.564\left(\frac{S_{eff} + 2}{32}\right)P_{LL} \tag{6-10}$$

where—

M_{LL} = *live-load bending moment of slab, in kip-feet per foot of width*
S_{eff} = *effective span length, in feet (equation 6-4)*
P_{LL} = *critical concentrated live load per axle, in kips (equation 6-9)*

6-29. **Required Nominal Strength.** Compute the required nominal strength as follows:

$$m = \frac{M_{DL} + M_{LL}}{0.9} \tag{6-11}$$

where—

m = *required nominal strength, in kip-feet per foot of width*
M_{DL} = *dead-load bending moment of the slab, in kip-feet per foot of width (equation 6-8)*
M_{LL} = *live-load bending moment of slab, in kip-feet per foot of width (equation 6-10)*

6-30. **Reinforcing Steel Ratio.** For a concrete compressive strength of 3,000 psi, use 0.85 for the B-factor to find the maximum reinforcement ratio in *Table 6-5*. Use the concrete compressive strength ($f'c$) and the B-factor to find the reinforcing steel ratio from *Table 6-5*.

Table 6-5. Reinforcing Steel Ratio (R_S)

Yield Strength (f_y)	f′c = 3,000 psi B = 0.85	f′c = 3,500 psi B = 0.85	f′c = 4,000 psi B = 0.85	f′c = 5,000 psi B = 0.80	f′c = 6,000 psi B = 0.75
40,000 psi	0.0139	0.0163	0.0186	0.0219	0.0246
50,000 psi	0.0103	0.0121	0.0138	0.0162	0.0182
60,000 psi	0.0080	0.0094	0.0107	0.0126	0.0142
Note: $R_s = 0.375R_{bs}$					

6-31. **Strength Coefficient of Resistance.** Compute the strength coefficient of resistance as follows:

$$R_n = R_s f_y - \left(\frac{R_s^2 f_y^2}{1.7f'c}\right) \tag{6-12}$$

where—

R_n = *strength coefficient of resistance, in psi*
R_S = *selected reinforcing steel ratio for design (Table 6-5)*
f_y = *yield strength of reinforcing steel, in psi (Table 6-3, page 6-9)*
f'_c = *compressive strength of concrete, in psi (Table 6-2, page 6-9)*

6-32. **Effective Depth.** Assuming that number (No.) 6 steel bars are used (nominal diameter of the bar equals 3/4 inch plus an additional 3/4 inch for protective concrete cover) and that the bars will fit in one layer, compute the required overall depth as follows:

$$req\ h = \sqrt{\frac{1{,}000m}{R_n}} + 1.125 \qquad (6\text{-}13)$$

where—

req h = required overall depth, in inches
m = required nominal strength, in kip-feet per foot of width (equation 6-11)
R_n *= strength coefficient of resistance, in psi (equation 6-12)*

6-33. Increase the required overall depth by about 1/2 inch (or by an amount that will round the required overall depth to the next complete inch or half-inch, whichever is closest). This will become the final thickness of the concrete deck. Compute the effective depth as follows:

$$d' = \sqrt{\frac{1{,}000m}{R_n}} + 0.5 \qquad (6\text{-}14)$$

where—

d′ = effective depth, in inches
m = required nominal strength, in kip-feet per foot of width (equation 6-11)
R_n *= strength coefficient of resistance, in psi (equation 6-12)*

6-34. **Revised Reinforcing Steel Ratio.** Compute the required area of tension steel to be placed in the transverse direction of the slab. Compute the revised value of the reinforcing steel ratio and the required steel area as follows:

$$R_s = \frac{0.85f'c}{f_y}\left[1 - \sqrt{1 - \frac{2{,}350m}{f'c(d')^2}}\right] \qquad (6\text{-}15)$$

where—

R_S *= revised reinforcing steel ratio*
f'_c *= compressive strength of concrete, in psi (Table 6-2, page 6-9)*
f_y *= yield strength of reinforcing steel, in psi (Table 6-3, page 6-9)*
m = required nominal strength, in kip-feet (equation 6-11)
d′ = effective depth, in inches (equation 6-14)

and—

$$A_{st} = 12R_s d' \qquad (6\text{-}16)$$

where—

A_{st} *= required area of tension steel, in square inches*
R_S *= revised reinforcement ratio (equation 6-15)*
d′ = effective depth, in inches (equation 6-14)

6-35. **Bar Selection and Placement.** Select the actual number of bars that will meet the tension steel area *(equation 6-16)* using *Table 6-6*. Use at least two bars wherever flexural reinforcement is required. Do not use more than two bar sizes at a given location in the span. The selected bars should not be more than two standard sizes apart (for example, No. 7 and No. 9 bars may be acceptable, but No. 4 and No. 9 would not).

Table 6-6. Total Areas for Various Numbers of Reinforcing Bars

Bar-Size No.	Nominal Diameter (in)	Weight (lb/ft)	Number of Bars									
			1	2	3	4	5	6	7	8	9	10
3	0.375	0.376	0.11	0.22	0.33	0.44	0.55	0.66	0.77	0.88	0.99	1.10
4	0.500	0.668	0.20	0.40	0.60	0.80	1.00	1.20	1.40	1.60	1.80	2.00
5	0.625	1.043	0.31	0.62	0.93	1.24	1.55	1.86	2.17	2.48	2.79	3.10
6	0.750	1.502	0.44	0.88	1.32	1.76	2.20	2.64	3.08	3.52	3.96	4.40
7	0.875	2.044	0.60	1.20	1.80	2.40	3.00	3.60	4.20	4.80	5.40	6.00
8	1.000	2.670	0.79	1.58	2.37	3.16	3.95	4.74	5.53	6.32	7.11	7.90
9	1.128	3.400	1.00	2.00	3.00	4.00	5.00	6.00	7.00	8.00	9.00	10.00
10	1.270	4.303	1.27	2.54	3.81	5.08	6.35	7.62	8.89	10.16	11.43	12.70
11	1.410	5.313	1.56	3.12	4.68	6.24	7.80	9.36	10.92	12.48	14.04	15.60

6-36. Locate the bars symmetrically about the vertical axis of the beam section (in one layer if practical). Select a bar size so that no less than two and no more than five or six bars are put in one layer. When using several layers of different bar sizes, place the largest bars in the layer nearest to the face of the beam. When placing bars within the beam's width, follow these guidelines for determining the minimum clear spacing required between the bars that will allow for proper concrete placement around them:

- For one layer of bars, the minimum clear spacing is 1 inch or the nominal diameter of the larger bar *(Table 6-6)*, whichever is greater.
- For two or more layers of bars, the minimum clear spacing is equal to or greater than 1 inch.

6-37. Ensure that the bar spacing obtained with *equation 6-16* is greater than the minimum clear spacing obtained previously with *equation 6-15*. Compute the actual spacing between the bars as follows:

$$\textit{bar spacing (in)} = \frac{10.5 - (\textit{number of bars x } d_b)}{\textit{total number of bars - 1}} \qquad (6\text{-}17)$$

where—

number of bars = *number of reinforcing bars selected from Table 6-6 that will accommodate the total area of steel (equation 6-16)*

d_b = *nominal diameter of the bar, in inches (Table 6-6)*

total number of bars = *total number of bars to be accommodated within the beam's width*

6-38. **Design Check.** Compute the depth of the equivalent rectangular stress block *(equation 6-18)* and then the design strength of the section *(equation 6-19)*. First compute—

$$d_o = \frac{A_{st} f_y}{10.2 f'c} \tag{6-18}$$

where—

d_o = *depth of the equivalent rectangular stress block, in inches*
A_{st} = *required area of tension steel, in square inches (equation 6-16)*
f_y = *yield strength of reinforcing steel, in psi (Table 6-3, page 6-9)*
f'_c = *compressive strength of concrete, in psi (Table 6-2, page 6-9)*

then compute—

$$m' = \left[A_{st} f_y \left(d' - \frac{d_o}{2}\right)\right] \frac{1}{12{,}000} \tag{6-19}$$

where—

m' = *design strength of the section, in kip-feet*
A_{st} = *required area of tension steel, in square inches (equation 6-16)*
f_y = *yield strength of reinforcing steel, in psi (Table 6-3)*
d' = *effective depth, in inches (equation 6-14)*
d_o = *depth of the equivalent rectangular stress block, in inches (equation 6-18)*

6-39. The section is acceptable in flexure if m' from *equation 6-19* is greater than or equal to m from *equation 6-11*. If the condition is not satisfied, go back to *equation 6-14* and increase the effective depth of slab slightly and redo all the necessary calculations until the condition is satisfied.

6-40. **Temperature and Shrinkage.** Reinforcing bars (parallel to traffic) are required in the top of the slab. Use the following guidelines for computing the minimum temperature reinforcement ratio (denoted by R_{temp}):

- The R_{temp} is 0.0020 when using grade 40 or 50 bars for the slabs.
- The R_{temp} is 0.0018 when using grade 60 bars for the slabs.

6-41. Compute the area of temperature and shrinkage steel as follows:

$$A_{temp} = 12 R_{temp} d' \tag{6-20}$$

where—

A_{temp} = *area of temperature and shrinkage steel, in square inches*
d' = *effective depth, in inches (equation 6-14)*
R_{temp} = *minimum temperature reinforcement ratio*

Place the temperature reinforcement bars at a minimum spacing equal to three times the slab thickness. Do not exceed a spacing of 18 inches.

6-42. **Shear Check.** Because of practical space limitations, shear reinforcement is not used in a slab. Slabs designed for bending moment should be considered satisfactory in shear.

Curbs and Handrails

6-43. A *curb system* guides traffic on the bridge. For timber-deck bridges, place 6- x 6-inch timbers on 5-foot centers on the curb risers. Risers of 6- x 12- x 30-inch material provide an adequate curb system. Rigidly attach the curbs to the decking. *Figure 6-5* shows the minimum specifications for a curb system on timber-deck bridges. The curb system will not withstand the impact of a heavy vehicle that is out of control. To design such a system would require a curb system of excessive size and cost. For concrete-deck bridges, form the curb as a part of the deck. Pour the curb at the same time as the main deck. Provide drain holes in the curb at 10-foot intervals on both sides of the bridge.

6-44. Include a *handrail system* if necessary. Place handrails on bridges with heavy foot traffic and where the danger of falling exists. If handrails are not used, mark the bridge edges. One marking method is to place 2- x 4-inch posts with reflectors at 10-foot intervals along both sides of the bridge. *Figure 6-5* shows the minimum specifications for handrails on timber-deck bridges.

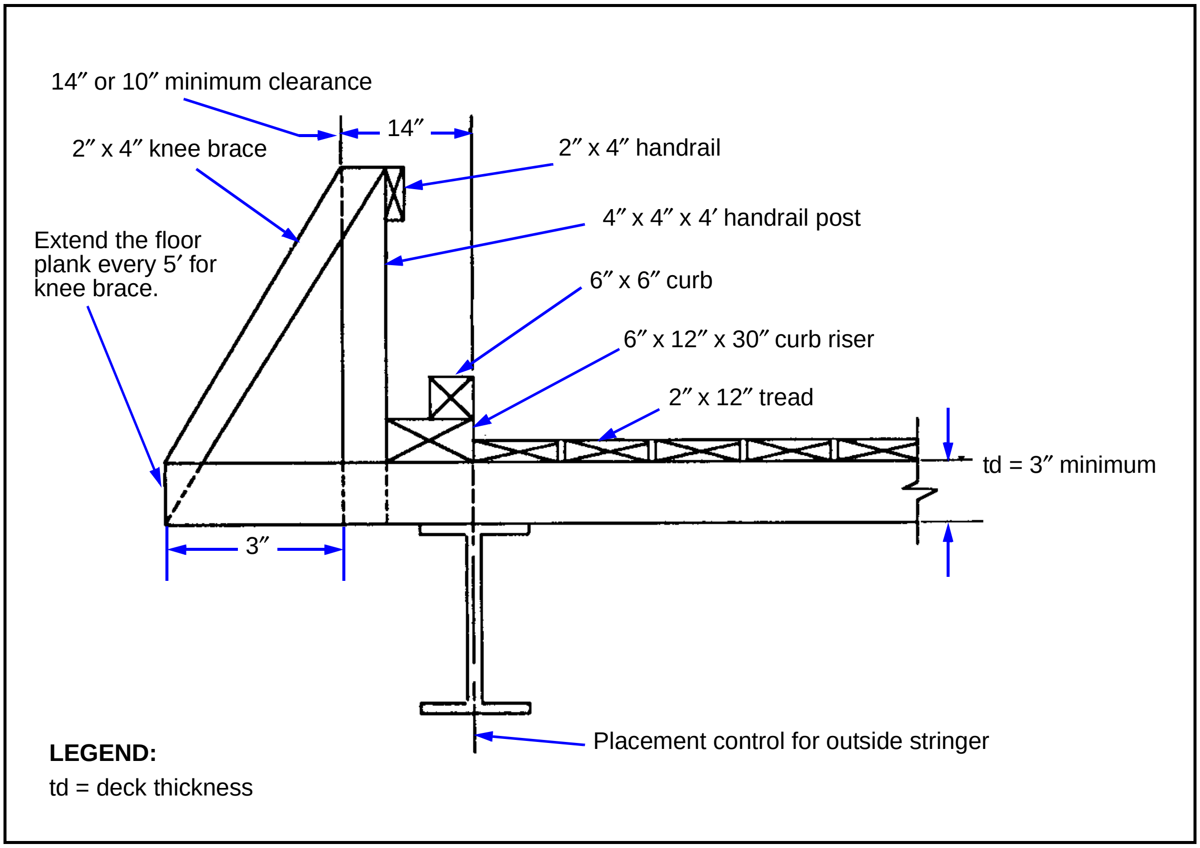


Figure 6-5. Curb and Handrail Systems for Timber-Deck Bridges

STRINGER DESIGN

6-45. Stringer design involves computing the dead-load weight, the live-load moment, and the total design moment. Various factors must be considered in the stringer selection (timber or steel).

Dead Load

6-46. The dead load includes the weight of all the parts of a structure, including the deck and accessories (railings, curbs, lateral bracing, and connections) as well as the stringers. Since the stringers are not yet sized, their weight must be estimated. The dead load is considered to be uniformly distributed along the span and equally shared by each stringer.

6-47. For the initial design calculations on a timber-deck bridge, assume a dead-load weight of 0.1 kpf for any accessories and a weight equal to 0.2 kpf per stringer. For a concrete-deck bridge, assume a dead-load weight of 0.4 kpf for any accessories (includes curbs and handrails) and a weight equal to 0.3 kpf per stringer. Compute the dead-load weight of the deck *(equation 6-21)* and the estimated design dead load *(equation 6-22)*. First compute:

$$w_{DL} = \left(\frac{b_s U_m t_s}{12{,}000}\right) + \left(\frac{b_R U_w t_w}{12{,}000}\right) \tag{6-21}$$

where—

w_{DL} = *dead-load weight of the deck, in kpf*

b_S = *deck width in the transverse direction, in feet*

U_m = *unit weight of the material, in pounds per cubic foot (Table 6-4, page 6-10)*

t_S = *timber deck or concrete-slab thickness, in inches*

b_R = *curb-to-curb width, in feet*

U_w = *unit weight for wearing-surface material, in pounds per cubic foot (Table 6-4)*

t_w = *wearing-surface thickness, in inches*

then compute—

$$W_{DL} = w_{DL} + w_{acc} + w_s N_s \tag{6-22}$$

where—

W_{DL} = *estimated design dead load of the span, in kpf*

w_{DL} = *dead-load weight of the deck, in kpf (equation 6-21)*

w_{acc} = *assumed dead-load weight of the accessories, in kpf*

w_S = *assumed dead-load weight per stringer, in kpf (paragraph 6-47)*

N_S = *total number of stringers (equation 6-1)*

6-48. Assume that the dead load of the bridge is equally shared by all of the stringers. Compute the design dead-load moment per stringer as follows:

$$m'_{DL} = \frac{W_{DL} L^2}{8 N_s} \tag{6-23}$$

where—

m'_{DL} = *design dead-load moment per stringer, in kip-feet*

W_{DL} = *estimated design dead load of the span, in kpf (equation 6-22)*

L = *design span length, in feet*

N_S = *total number of stringers (equation 6-1)*

Live Load

6-49. Vehicle loads are assumed to be the only live load acting on the bridge. Find the design values for the live load by using the moment and shear curves in *Appendix B*. Use the larger value of wheeled and tracked moment and both values of wheeled and tracked shear for further calculations. If the bridge is for civilian traffic, use the provisions in *Chapter 3, Section III*, and *Figure 3-1, page 3-4*, to determine the equivalent MLC of the civilian traffic for design calculations.

6-50. NATO traffic restrictions apply for design purposes, which is 25 mph and 100-foot spacing. Because of this long spacing, usually only one vehicle will be on any single span of a bridge at a time. If significant pedestrian traffic is expected (refugees and dismounted military units), treat these as line loads of 75 pounds per foot, each over a 1-foot width. Post these line loads in all locations that lines of people might be expected.

6-51. Determine the total live-load moment according to the design vehicle class *(Table B-2, pages B-6* through *B-9)*. Compute the live-load moment that a stringer must resist (including impact effects) as follows:

$$m'_{LL} = \frac{1.15M'_{LL}}{N_{1,2}} \tag{6-24}$$

where—

m'_{LL} = *design live-load moment per stringer, in kip-feet*

M'_{LL} = *total design live-load moment according to vehicle class, in kip-feet (Table B-2)*

$N_{1,2}$ = *effective number of stringers (Table 3-3, page 3-14)*

Total Design Moment per Stringer

6-52. The total design moment that each stringer resists is the summation of the dead- and live-load moments per stringer. Compute as follows:

$$M = m'_{DL} + m'_{LL} \tag{6-25}$$

where—

M = *total design moment per stringer, in kip-feet*

m'_{DL} = *design dead-load moment per stringer, in kip-feet (equation 6-23)*

m'_{LL} = *design live-load moment per stringer, in kip-feet (equation 6-24)*

Timber-Stringer Selection

6-53. If the species and grade of timber is known, use the allowable stresses from *Table C-1, pages C-3* through *C-6*, for the design. Convert these values from psi to ksi by dividing the tabulated stress by 1,000. Generally, the tabulated values assume that the material will be in continuously dry conditions. However, timber decking and stringers may retain moisture on their horizontal surfaces; therefore, consider its use under wet conditions.

6-54. Apply some modification factors (see the notes in *Table C-1, pages C-3* through *C-6,* toward the tabulated allowable bending stress to account for various effects (lumber thickness/width ratios, edgewise or flatwise use, repetitive member use, and moisture content). For military loads, apply an additional factor equal to 1.33 to account for lower traffic volume (see *Appendix I*). Whenever the species and grade of solid-sawn timber cannot be determined, assume an allowable bending stress equal to 1.75 ksi and an allowable horizontal-shear stress equal to 0.095 ksi. For glue-laminated timber, assume an allowable bending stress equal to 2.16 ksi and an allowable horizontal-shear stress equal to 0.2 ksi. These values must be adjusted for the various applicable conditions stated in the notes of *Table C-1*. The modification for lower traffic volume has already been considered in those assumed values.

Steel-Stringer Selection

6-55. The allowable bending stress for steel members is 0.75 times the steel yield strength *(Table 6-7)*, assuming that the stringers are braced properly *(paragraph 6-59)*. The allowable shear stress for steel members is 0.45 times the steel yield strength.

Table 6-7. Steel Yield Strength

Type	Structural Steel	High-Strength, Low-Alloy Steel		Quenched and Tempered, Low-Alloy Steel	High Yield Strength, Quenched and Tempered, Alloy Steel	
AASHTO designation	M 270 (Grade 36)	M 270 (Grade 50)	M 270 (Grade 50)	M 270 (Grade 70W)	M 270 (Grades 100/100W)	
ASTM designation	A 709 (Grade 36)	A 709 (Grade 50)	A 709 (Grade 50W)	A 709 (Grade 70W)	A 709 (Grade 100/100W)	
Plate thickness	Up to 4 in	Up to 4 in	Up to 4 in	Up to 4 in	$\leq$ 2 1/2 in	$>$ 2 1/2 to $\leq$ 4 in
F_y (ksi)	36	50	50	70	100	90

Required Section Modulus

6-56. Once the total moment each stringer must resist is known, compute the section modulus a stringer requires for a given allowable bending stress as follows:

$$S_{req} = \frac{12M}{F_b} \tag{6-26}$$

where—

S_{req} = *required section modulus, in cubic inches*

M = *total design moment per stringer, in kip-feet (equation 6-25)*

F_b = *allowable bending stress, in ksi (paragraph 6-53 for timber and paragraph 6-55 for steel)*

6-57. Select a stringer *(Table C-2, page C-7,* for timber and *Table D-2, pages D-3* and *D-4,* for steel*)* with a section modulus greater than or equal to the required section modulus from *equation 6-26*. The available stringer sizes may not be large enough to provide sufficient section modulus. If this happens, add a stringer to the bridge section and recompute *(equation 6-23)*. Compute the total design moment per stringer *(equation 6-25)* and the required section modulus *(equation 6-26)* until a suitable steel stringer size (as listed in *Appendix D)* is obtained.

VERTICAL-DEFLECTION CHECK

6-58. Compute the vertical deflection of the stringers due to the live load (including impact) as shown below. The deflection should not be greater than $d_{max} = (L/200) \times 12$ (in inches).

$$d_{LL} = \frac{331M'_{LL}L^2}{N_{1,2}ESd_s} \qquad (6\text{-}27)$$

where—

d_{LL} = *deflection due to the live load plus impact, in inches*

M'_{LL} = *total design live-load moment according to vehicle class, in kip-feet (Table B-2, pages B-6 through B-9)*

L = *design span length, in feet*

$N_{1,2}$ = *effective number of stringers from Table 3-3, page 3-14*

E = *modulus of elasticity, in ksi (Appendix C for timber and Appendix D for steel)*

S = *section modulus of the selected stringer, in cubic inches*

d_S = *depth of the stringer, in inches*

LATERAL-BRACING DESIGN

6-59. When a beam is loaded and deflected downward, the upper portion of the beam shortens and the lower portion of the beam lengthens. This reshaping results from the internal moments induced by the loading. The beam will experience compressive forces in the upper portion and tensile forces in the lower portion of the section. The upper portion of the member tends to compress or buckle, just as a column does with respect to its weaker axis. The buckling effect is always accompanied by some lateral twisting. This action is called lateral buckling. *Figure 6-6, page 6-20,* shows the lateral-buckling effect in a beam (timber or steel).

6-60. To prevent lateral buckling in a beam, use cross frames or diaphragms and bracing systems for lateral support. One of the primary factors affecting lateral-beam stability is the distance between the points of a lateral support along the beam's length (the unsupported or unbraced length).

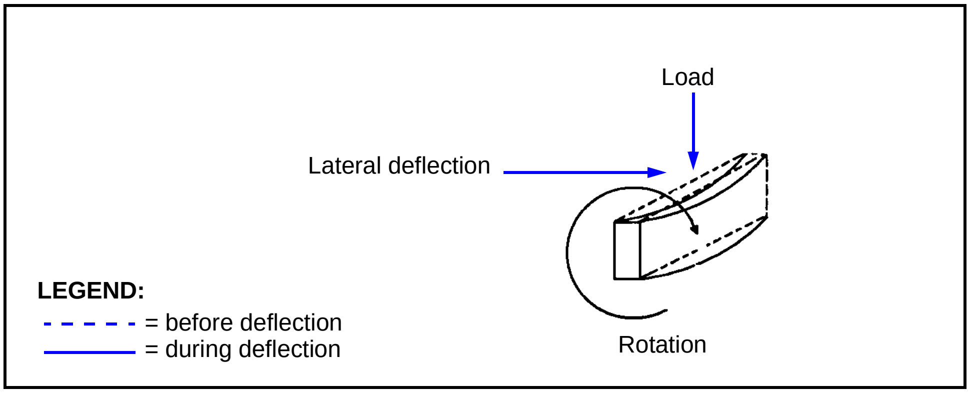


Figure 6-6. Lateral Buckling

Timber Stringers

6-61. For timber beams, provide lateral support by locating transverse bracing at the beam's end supports and at every one-third point along the beam span *(Figure 6-7)*. This distance (spacing) between the lateral braces along the length of the beam is the unbraced length. For simple-span beams and any loading condition, compute the effective beam length and then the beam slenderness factor as follows:

$$\text{If } \frac{12L_u}{d_s} \text{ is} \geq 14.3\text{, use } L_e = 22.1L_u \tag{6-28}$$

or—

$$\text{If } \frac{12L_u}{d_s} \text{ is} < 14.3\text{, use } L_e = 19.6L_u + 3d_s \tag{6-29}$$

where—

L_u = *unbraced length, in inches*
d_S = *depth of the timber stringer, in inches*
L_e = *effective beam length, in inches*

$$C_s = \sqrt{\frac{L_e d_s}{b_s^{\,2}}} \leq 50 \tag{6-30}$$

where—

C_S = *slenderness factor (nondimensional)*
L_e = *effective beam length, in inches (equation 6-28 or 6-29)*
d_S = *depth of the timber beam, in inches*
b_S = *stringer width, in inches*

6-62. If the result of *equation 6-30* is not ≤50, increase the number of braces along the beam's span to reduce the unsupported length and then recompute. When the unbraced length varies substantially along the beam's span, check the slenderness factor for each unsupported length. Typically, the slenderness

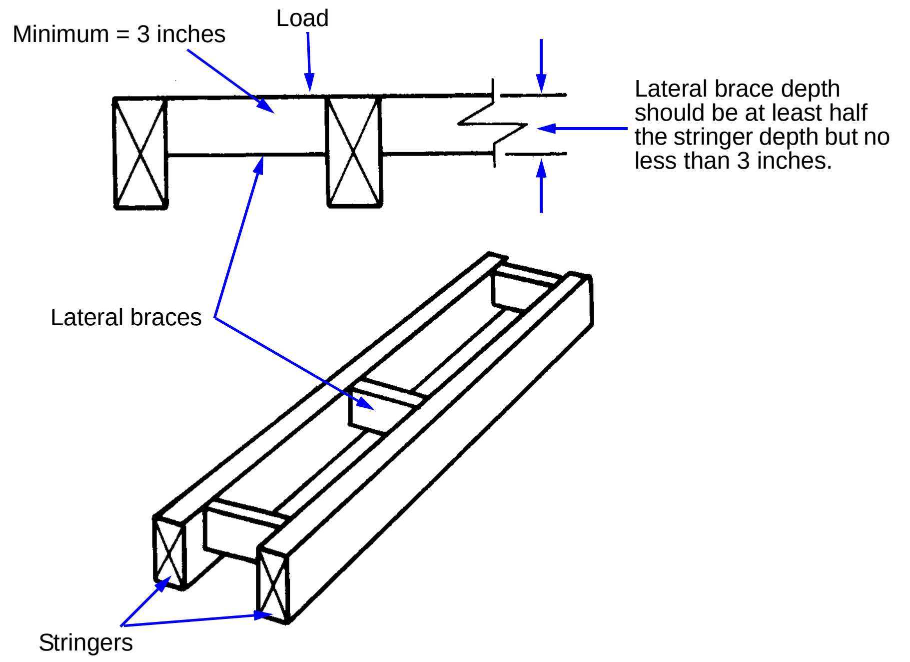


Figure 6-7. Lateral Bracing for Timber Stringers

factor at the center portion of the beam (where bending stress is higher) will control the lateral-bracing design.

Steel Stringers

6-63. A steel beam should be braced laterally, perpendicular to the plane of the web. Lateral bracing provides adequate lateral stability of the compression flange so that the beam section can develop its maximum design bending strength.

6-64. **Maximum Allowable Unbraced Length.** Establish the maximum allowable unbraced length for a steel stringer by using the smaller of the values obtained from the following equations:

$$L_c = \frac{6.33 b_f}{\sqrt{F_y}} \tag{6-31}$$

or—

$$L_c = \frac{1{,}667}{\left(\frac{d_s}{A_f}\right) F_y} \tag{6-32}$$

where—

L_c = *maximum allowable unbraced length for a steel stringer, in feet*
b_f = *flange width of the steel section, in inches (Appendix D)*

F_y = *yield strength of steel, in ksi (Table 6-7, page 6-18)*
d_S = *depth of the steel section, in inches (Appendix D)*
A_f = *area of the compression flange, in square inches (Appendix D)*

6-65. **Number of Braces.** The number of lateral braces needed will depend on the length of the beam's span and the maximum unbraced length for a given steel section. Compute for the number of lateral braces needed as follows:

$$N_b = \frac{L}{L_c} + 1 \qquad (6\text{-}33)$$

where—

N_b = *number of lateral braces (rounded to the next higher whole number)*
L = *design span length, in feet*
L_c = *maximum allowable unbraced length for a steel stringer, in feet (the smaller of the value's from equations 6-31 and 6-32)*

6-66. **Spacing of Lateral-Bracing.** Distribute the number of lateral braces by spacing them along the beam span at the distance computed below. Locate a lateral brace at each end support and the remaining braces along the beam's length.

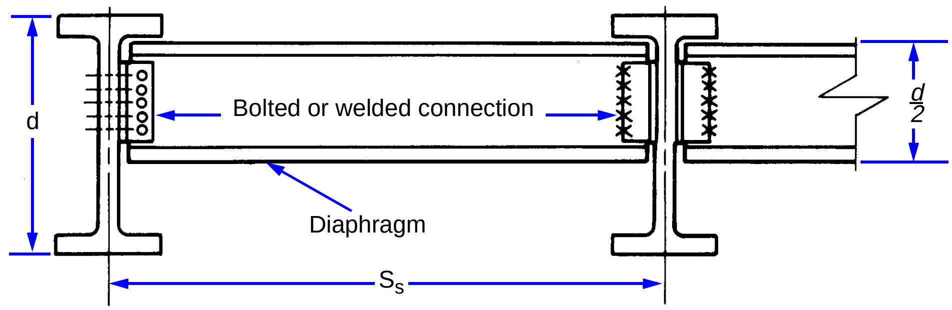
$$L_u = \frac{L}{N_b - 1} \qquad (6\text{-}34)$$

where—

L_u = *spacing of lateral bracing , in feet*
L = *design span length, in feet*
N_b = *number of lateral braces (rounded to the next higher whole number) (equation 6-33)*

6-67. **Bracing-System Selection.** The type of lateral bracing depends on the availability of materials. Diaphragms or cross frames are satisfactory braces. Diaphragms are generally more economical for rolled shapes that are less than 32 inches deep. Cross frames are generally more economical for built-up beams that are 32 inches and deeper.

6-68. Diaphragms are rolled shapes used in a lateral-bracing system *(Figure 6-8)*. The diaphragm depth should be at least half the depth of the steel stringer. Construct diaphragms from the lightest materials available. Although the most suitable diaphragms are constructed using channel sections, any rolled shape (such as an I-beam) is satisfactory. Precut ends of stringers are available for fabricating diaphragms. Structural Ts can be used as diaphragms. Form these shapes by cutting the excess stringer material in half, along the centerline of the web. When using structural Ts, place the flange as close as possible to the stringer's compression flange (which is the top flange) and weld the connections. A bolted connection (with 3/4-inch or 7/8-inch bolts at a minimum spacing along a single row) may also be used. See *Chapter 9* for more information.



LEGEND:
d = depth of stringer
S_s = center-to-center stringer spacing

Figure 6-8. Highway-Bridge Diaphragm

6-69. Cross frames are used when the depth of the stringer exceeds 32 inches. Use equal leg angles configured into a cross frame as a more economical alternative to using diaphragms *(Figure 6-9)*. However, the increased cost of cutting and fabrication outweighs any material savings. Minimum requirements of the angles are that—

- The dimensions of the member should not be smaller than 3 x 3 x 3 1/8 inches.
- The thickness of the member should be greater than one-tenth the length of the longer leg.
- The ratio of the span length to the radius of gyration of the section used for bracing *(12L/r,* where L is in feet and r is in inches) must be less than or equal to 200.

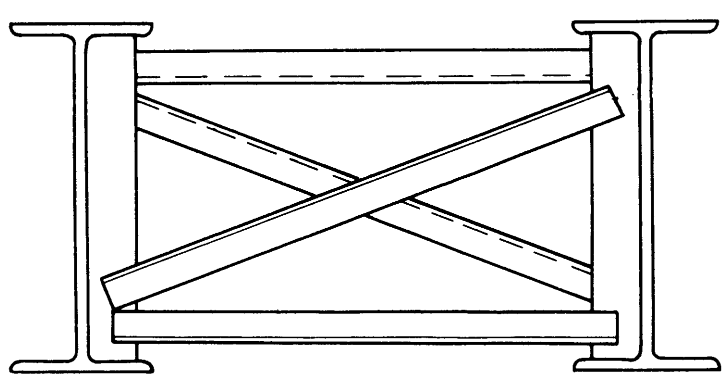


Figure 6-9. Cross-Frame Beam Bracing

DEAD-LOAD CHECK

6-70. After designing the deck and selecting the stringer size, check the initial dead-load assumption for any necessary corrections. The total dead load per stringer consists of the combined dead loads of the deck system, the stringers, the lateral bracing, and any accessories. Any changes in the total dead-load value may result in an increase or decrease of the required section modulus.

Component Loads

6-71. Compute the dead load for the deck as follows:

$$w_d = \frac{b_s U_m t_s}{12{,}000} \tag{6-35}$$

where—

w_d = *dead load due to the deck, in kpf*
b_S = *slab width in the transverse direction, in feet (usually* $b_R + 4$*)*
U_m = *unit weight of the material, in pounds per cubic foot (Table 6-4, page 6-10)*
t_S = *slab thickness, in inches*

6-72. Compute the dead load for the wearing surface as follows:

$$w_w = \frac{b_R U_w t_w}{12{,}000} \tag{6-36}$$

where—

w_w = *dead load due to the wearing surface, in kpf*
b_R = *curb-to-curb width, in feet*
U_w = *unit weight for wearing-surface material, in pounds per cubic foot (Table 6-4)*
t_w = *wearing-surface thickness, in inches*

6-73. Compute the dead load for the stringers as follows:

$$w_s = \frac{A_t U_m N_s}{144{,}000} \tag{6-37}$$

where—

w_S = *weight of the timber stringers, in kpf*
A_t = *cross-sectional area of one timber stringer, in square inches*
U_m = *unit weight of the material, in pounds per cubic foot (Table 6-4)*
N_S = *total number of stringers in the span*

6-74. For steel stringers, the last number in the nomenclature of a steel section corresponds to its weight (in pounds per foot). For example, as shown in *Appendix D*, a W27x94 would indicate that this section weighs 94 pounds per foot. Compute the weight due to the steel stringers as follows:

$$w_s = \frac{W_s N_s}{1{,}000} \tag{6-38}$$

where—

w_S = *weight of steel stringers, in kpf*
W_S = *weight of the steel section, in pounds per foot (Appendix D)*
N_S = *total number of stringers in the span*

6-75. Compute the dead load due to accessories as follows:

$w_{acc} = 0.1$ for timber-deck bridges (6-39)

or—

$w_{acc} = 0.4$ for concrete-deck bridges (6-40)

where—

w_{acc} = *dead load due to the accessories, in kpf*

6-76. Compute the length of the lateral bracing as follows:

$$L_b = (N_s - 1)\left(S_s - \frac{t_{ws}}{12}\right) \quad (6\text{-}41)$$

where—

L_b = *length of the lateral braces, in feet*
N_S = *number of stringers in the transverse direction of the bridge*
S_S = *actual center-to-center stringer spacing, in feet (equation 6-2)*
t_{ws} = *thickness of the web of the stringer, in inches*

and—

$$w_b = \frac{(N_b - 2)L_b U_m}{L} \quad (6\text{-}42)$$

where—

w_b = *weight of the lateral braces, in kpf*
N_b = *total number of lateral braces in the span*
L_b = *length of the lateral braces, in feet (equation 6-41)*
U_m = *unit weight of the lateral-bracing material, in kpf (the last number in the nomenclature is the kpf for steel and 0.1 kpf for timber)*
L = *design span length, in feet*

Actual Dead Loads

6-77. Compute the total actual dead load as follows:

$$W'_{DL} = w_d + w_w + w_s + w_{acc} + w_b \quad (6\text{-}43)$$

where—

W'_{DL} = *total actual dead load, in kpf*
w_d = *dead load due to the deck, in kpf (equation 6-35)*
w_w = *dead load due to the wearing surface, in kpf (equation 6-36)*

w_S = *weight of the stringer, in kpf (equation 6-37 for timber and equation 6-38 for steel)*

w_{acc} = *dead load due to the accessories, in kpf (equation 6-39 for timber-deck bridges and equation 6-40 for concrete-deck bridges)*

w_b = *weight of the lateral braces, in kpf (equation 6-42)*

6-78. Compute the actual dead load carried per stringer as follows:

$$w'_{DL} = \frac{W'_{DL}}{N_s} \tag{6-44}$$

where—

w'_{DL} = *actual dead load carried per stringer, in kpf*

W'_{DL} = *total actual dead load, in kpf (equation 6-43)*

N_S = *total number of stringers in the span*

6-79. Compare the actual dead load to the estimated design dead load per stringer as follows:

$$w'_{DL} \leq \frac{W_{DL}}{N_s} \tag{6-45}$$

where—

w'_{DL} = *actual dead load carried per stringer, in kpf (equation 6-44)*

W_{DL} = *estimated design dead load, in kpf (equation 6-22)*

N_S = *total number of stringers in the span*

6-80. If the results of *equation 6-45* work, then the selected bridge components are considered adequate based on the estimated dead load. Complete the design and classify the bridge as described in *Chapter 3*. If the results of *equation 6-45* do not work, adjust the stringer size by redoing all the necessary calculations as follows:

- Replace the estimated design dead load *(equation 6-22)* with the total actual dead load *(equation 6-43)*.
- Recompute all related equations to obtain a new section modulus, and select a new stringer section.
- Check the dead-load requirements with the values and continue recomputing until the results of *equation 6-45* work.

SHEAR-FORCE CHECK

6-81. After selecting a stringer size, check the stringer's shear capacity. In most timber-stringer bridges with spans of less than 20 feet, shear controls the design. In steel-stringer bridges with a high design classification and short spans (20 feet or less), shear may be a critical factor.

Dead-Load Shear per Stringer

6-82. Assume that the design dead-load shear is equally distributed among all the stringers. Compute the design dead-load shear per stringer as follows:

$$v'_{DL} = \frac{w'_{DL}L}{2N_S} \tag{6-46}$$

where—

v'_{DL} = *design dead-load shear per stringer, in kips*
w'_{DL} = *actual dead load carried per stringer, in kpf (equation 6-44)*
L = *design span length, in feet*
N_S = *total number of stringers*

Design Live-Load Shear

6-83. Determine the design live-load shear. Use *Table B-3, pages B-10 through B-13.*

6-84. **Effective Live-Load Shear per Stringer.** The effective live-load shear must account for the loads at the abutments or intermediate supports and for those further out on the span. Since steel-stringer bridges act very similarly to glue-laminated timber bridges, the equations used to compute the effective live-load shear per stringer are the same. The live-load shear per stringer corresponds to the largest value from the following equations:

- **Wheeled vehicle, one traffic lane.**

$$v_{LL} = \left(0.5 + \frac{S_s}{32}\right)V_A + \left(\frac{V'_{LL} - V_A}{N_{1,2}}\right) \qquad (6\text{-}47)$$

where—

v_{LL} = *effective live-load shear per stringer, in kips*
S_S = *actual center-to-center stringer spacing, in feet (equation 6-2)*
V_A = *heaviest axle load, in kips (Table B-1, pages B-2 through B-5, Column 4)*
V'_{LL} = *design live-load shear for wheeled vehicles, in kips (paragraph 6-83)*
$N_{1,2}$ = *effective number of stringers (Table 3-3, page 3-14)*

- **Wheeled vehicle, two or more traffic lanes.**

$$v_{LL} = \left(\frac{S_s - 2}{S_s}\right)V_A + \left(\frac{V'_{LL} - V_A}{N_{1,2}}\right) \qquad (6\text{-}48)$$

where—

v_{LL} = *effective live-load shear per stringer, in kips*
S_S = *actual center-to-center stringer spacing, in feet (equation 6-2)*
V_A = *heaviest axle load, in kips (Table B-1, Column 4).*
V'_{LL} = *design live-load shear for wheeled vehicles, in kips (paragraph 6-83)*
$N_{1,2}$ = *effective number of stringers (Table 3-3)*

- **Tracked vehicle, one traffic lane.**

$$v_{LL} = \frac{V'_{LL}}{2} \qquad (6\text{-}49)$$

where—

v_{LL} = *effective live-load shear per stringer, in kips*

V'_{LL} = *design live-load shear for tracked vehicles, in kips (paragraph 6-83)*

- **Tracked vehicle, two or more traffic lanes.**

$$v_{LL} = \left(\frac{S_s - 2}{S_s}\right)V'_{LL} \quad (6\text{-}50)$$

where—

v_{LL} = *effective live-load shear per stringer, in kips*
S_S = *actual center-to-center stringer spacing, in feet (equation 6-2)*
V'_{LL} = *design live-load shear for tracked vehicles, in kips (paragraph 6-83)*

6-85. **Design Live-Load Shear per Stringer.** Compute the design live-load shear per stringer as follows:

$$v'_{LL} = \left(\frac{L - 0.0833d_s}{L}\right)v_{LL} \geq 0.75v_{LL} \quad (6\text{-}51)$$

where—

v'_{LL} = *design live-load shear per stringer, in kips*
L = *design span length, in feet*
d_S = *depth of the steel stringer, in inches (Appendix D)*
v_{LL} = *effective live-load shear per stringer, in kips (the largest value of wheeled or tracked classification from equations 6-47 through 6-50)*

6-86. **Design Shear per Stringer.** Compute the design shear per stringer and the actual shear stress acting on the stringer as follows:

$$v = v'_{DL} + 1.15v'_{LL} \quad (6\text{-}52)$$

where—

v = *design shear per stringer, in kips*
v'_{DL} = *dead-load shear per stringer, in kips (equation 6-46)*
v'_{LL} = *design live-load shear per stringer, in kips (equation 6-51)*

and—

$$f'_v = \frac{3v}{2A_s} \leq F_v \quad (6\text{-}53)$$

where—

f'_v = *actual shear stress, in ksi*
v = *design shear per stringer, in kips (equation 6-52)*
A_S = *area of a stringer, in square inches (Appendix C for timber and Appendix D for steel)*
F_v = *allowable shear stress, in ksi (paragraph 6-53 for timber and paragraph 6-55 for steel)*

6-87. If the actual shear stress *(equation 6-53)* is less than or equal to the allowable shear stress, the stringer will not have to be adjusted. However, if the actual shear stress is greater than the allowable shear stress, select a larger stringer size that satisfies the shear strength and moment capacity requirements.

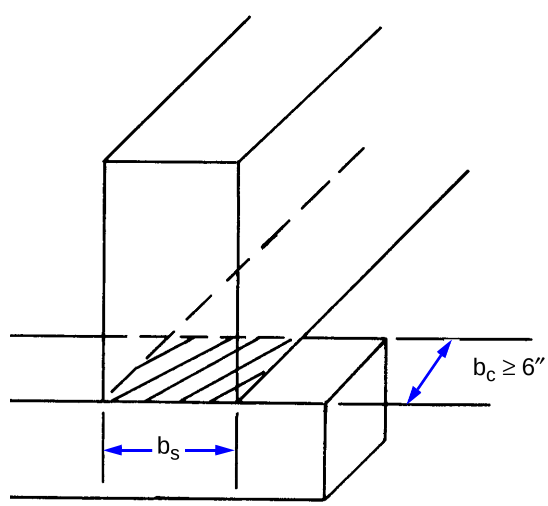
END-BEARING DESIGN (TIMBER STRINGERS)

6-88. Although bearing failure in timber stringers is rare, the bearing stress should still be checked. The minimum width of the cap (or sill) required for timber stringers is 6 inches *(Figure 6-10)*. Compute the actual bearing stress as follows:

$$f'_b = \frac{v}{b_s b_c} \tag{6-54}$$

where—

f'_b = *actual bearing stress, in ksi*
v = *design shear per stringer, in kips (equation 6-52)*
b_S = *width of the timber stringer, in inches*
b_c = *cap or sill width, in inches (which should be ≥ 6 inches)*



LEGEND:
b_c = width of cap
b_s = width of stringer

Figure 6-10. End-Bearing Timber Stringer

6-89. The actual bearing stress should not exceed the allowable bearing stress. The values for allowable bearing stress for timber are shown in *Table 6-8, page 6-30*. These design values are given for the wet- and dry-service conditions and were obtained as the average stress value from various species combinations. If the actual bearing stress exceeds the allowable bearing stress, then increase either the width of the cap or the width of the stringer to provide a sufficient bearing area.

Table 6-8. Allowable Bearing Stress for Timber

Service Conditions	Allowable Bearing Stress (ksi)
Sawn lumber (wet condition)	1.15
Sawn lumber (dry condition):	
5 x 5 inches or larger	1.32
2 x 4 inches thick	1.73
Glue-laminated (wet or dry)	2.04

END-BEARING DESIGN (STEEL STRINGERS)

6-90. Bearing plates are designed to transmit the loads from the superstructure into the substructure. If the bearing-seat area is insufficient to carry the load, failure will occur. Bearing failure causes the materials that bear together to crush, which may lead to stringer-flange failure. Another type of failure is web crippling (failure of the web portion of the stringer). Web crippling occurs due to the stress concentrations at the junction of the flange and the web where the beam is trying to transfer compression from a wide flange to a narrow web. *Figure 6-11* shows various types of end-bearing failures. Bearing plates are designed to prevent flange failure and crushing of the supports. Web crippling can be prevented by designing end-bearing stiffeners.

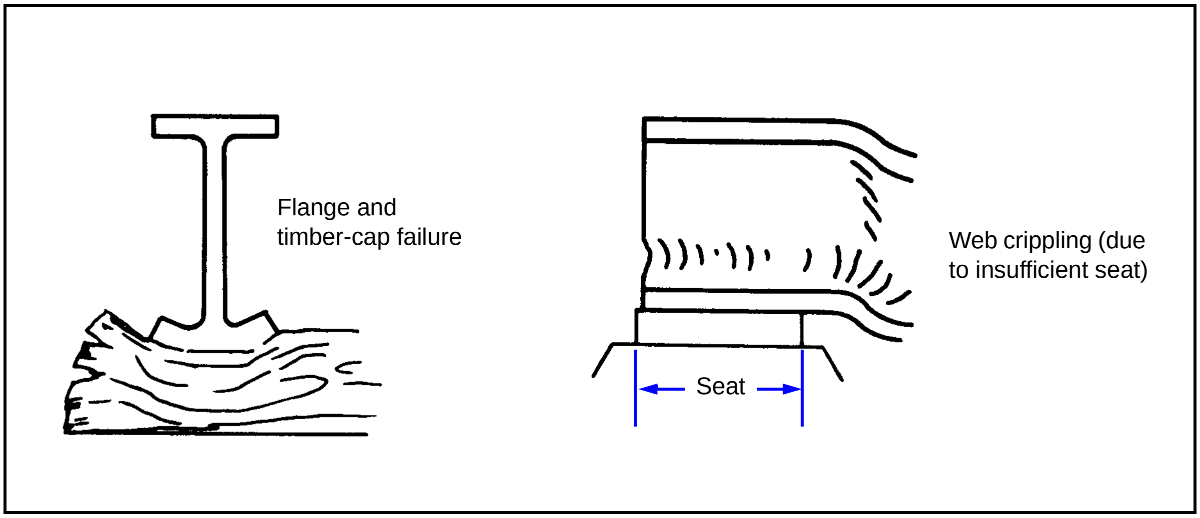


Figure 6-11. End-Bearing Failures

Bearing Plates

6-91. End-bearing plates are typically required when steel stringers rest on concrete or timber supports. Their design is based on the design shear transmitted to the support (adjusted if the actual dead load *[equation 6-45]* is greater than the estimated design dead load *[equation 6-22]*).

6-92. **Bearing-Plate Area.** The required bearing-plate area is determined by the allowable bearing stress of the support and the design shear that is carried to the support. Compute as follows:

$$A_{pl} = \frac{v}{F_B} \qquad (6\text{-}55)$$

where—

A_{pl} = *bearing-plate area, in square inches*
v = *design shear per stringer, in kips (equation 6-52)*
F_B = *allowable bearing stress of the support material, in ksi (equal to 0.75F_y)*

6-93. **Plate Width and Length.** The most economical plate design results when the seat width (which is the length of the plate) is increased while the plate width is minimized. The minimum width of the plate should be equal to the stringer-flange width. The minimum seat length is 6 inches. Therefore, the minimum plate area is six times the flange width.

6-94. **Plate Thickness.** Use these steps to determine the plate thickness.

Step 1. Compute the actual bearing stress as follows:

$$f'_B = \frac{v}{b_{pl} b_c} \qquad (6\text{-}56)$$

where—

f'_B = *actual bearing stress, in ksi*
v = *design shear per stringer, in kips (equation 6-52)*
b_{pl} = *plate width, in inches (paragraph 6-93)*
b_c = *cap or sill width, in inches (should be greater than or equal to 6 inches)*

Step 2. Compute the required thickness of the bearing plate as follows:

$$t_{pl} = \sqrt{\frac{3f'_B\left(\frac{b_{pl}}{2} - t_f\right)^2}{F_b}} \qquad (6\text{-}57)$$

where—

t_{pl} = *required plate thickness, in inches*
f'_B = *actual bearing stress, in ksi (equation 6-56)*
b_{pl} = *plate width, in inches (paragraph 6-93)*
t_f = *flange thickness, in inches*
F_b = *allowable bending stress, in ksi (paragraph 6-53 for timber and paragraph 6-55 for steel)*

Step 3. Select a bearing-plate thickness from the following available standard thicknesses:

- 1/32-inch increments (up to 1/2 inch).
- 1/16-inch increments (from 1/2 up to 1 inch).
- 1/8-inch increments (from 1 up to 3 inches).
- 1/4-inch increments (for 3 inches or greater).

Step 4. Build the bearing plate to the required thickness by laminating the plates. Fully weld the perimeters of the plate.

6-95. **Web Crippling.** Check for web crippling. If the actual bearing stress in the web exceeds the allowable bearing stress, use end-bearing stiffeners or increase the length of the bearing plate. Compute the actual bearing stress as follows:

$$f'_{bw} = \frac{v}{t_w(b_c + t_f)} \leq F_B \qquad (6\text{-}58)$$

where—

f'_{bw} = *actual bearing stress in the web, in ksi*
v = *design shear per stringer, in kips (equation 6-52)*
t_w = *web thickness, in inches*
b_c = *cap or sill width, in inches (should be greater than or equal to 6 inches)*
t_f = *flange thickness, in inches*
F_B = *allowable bearing stress, in ksi (equal to 0.9F_y)*

End-Bearing Stiffeners

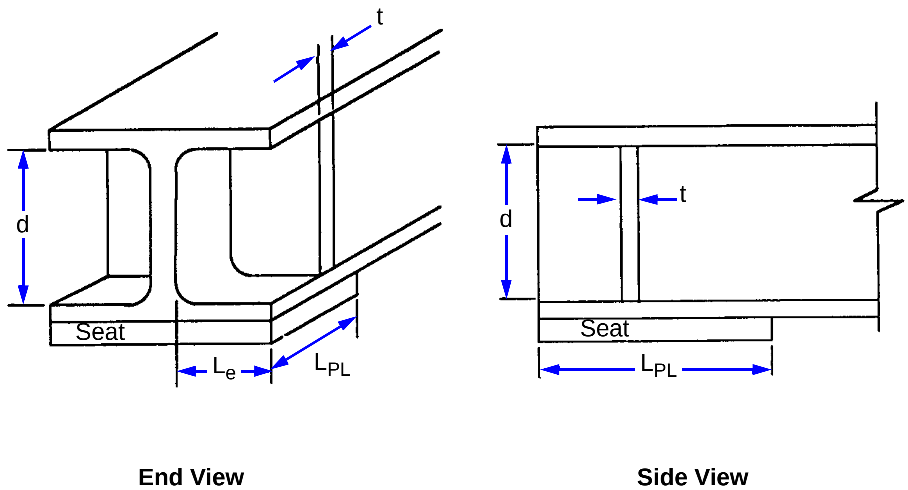
6-96. End-bearing stiffeners are normally not required for standard, rolled shapes unless abutment or intermediate-support dimensions restrict the length of the bearing plate. However, the web may have to be stiffened to prevent it from buckling. If end-bearing stiffeners are needed, construct them of angles or plates on each side of the stringer web. Position them over the center of the bearing at the end of each stringer. Ensure that they fit tightly against the flanges being loaded. Mill the top and bottom of the stiffener to bear against the flanges of the stringer, and extend them out as far as possible toward the edges of the flange *(Figure 6-12)*. Compute the minimum required thickness of the end-bearing stiffeners as follows:

$$t = \frac{L_e}{12}\sqrt{\frac{F_y}{33}} \qquad (6\text{-}59)$$

where—

t = *required thickness of end-bearing stiffeners, in inches*
L_e = *effective stiffener width, in inches (Figure 6-12)*
F_y = *yield strength of steel, in ksi (Table 6-7, page 6-18)*

6-97. Provide sufficient welding (or bolting) to transfer the total end shear through the web. Do not crimp the angles used as end stiffeners to fit over the flange angles. Instead, use filler plates between the web and the stiffeners.



LEGEND:

d = stiffener depth (web depth)
L_e = effective stiffener length
L_{PL} = seat length
t = stiffener thickness

Figure 6-12. End-Bearing Stiffener